Optimum Automated Direct Displacement Based Design Of Reinforced Concrete Frames

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SUMMARY:
Current seismic design of reinforced concrete frames include force based design for calculating equivalent lateral forces and static analysis, Research has revealed erroneous assumptions in force based design and proposes that displacement based design, due to modeling inelastic systems, result is more reasonable lateral force distributions. Another advantage is that force reduction factor outlined in current seismic codes is not required since the frame is designed for inelastic behavior. This approach eliminates discrepancies between initially assumed force reduction factors and final frame ductility capacity also individual member capacity checks outlined in design specifications are similarly not required. Displacement based design methods are emerging as the latest tool for performance based seismic design. This method allows design engineers to predict actual frame behavior with greater accuracy. Earthquake and structural engineering challenge of creating optimized, reliable and cost effective structures leads to the combination of optimization and performance based seismic design theory. The objective of this research is to automate the optimal design of the reinforced concrete frames that satisfy the limitations and specifications of the ACI code using a genetic algorithm (GA) and direct displacement based design method. In this paper automating the design process of reinforced concrete frames is performed by minimize the structural cost.

Keywords: performance based design, direct displacement based design, optimal design, genetic algorithm

1. INTRODUCTION

The objective of this research is to outline an alternate seismic engineering philosophy to the current codified force based design (FBD) approach for seismic resistant concrete moment frames known as Performance based Seismic Engineering (PBSE). Performance based engineering is any design methodology in which the final analytical outcome is measured against a performance limit state. These limit states can be quantitatively measured by forces, strains, rotations, or displacements, and are representative of member or system damage levels. It is the limit state form that delineates the design methodology into respective performance based groups. Since displacements are the most convenient practice of system and member evaluation, the analytical method employed in this research to achieve PBSE is Displacement based Design (DBD). Furthermore another objective of this research is to design low-cost reinforced concrete frames that satisfy the limitations and specifications of the American Concrete Institute (ACI) Building Code (ACI 318-02) and FEMA-356 (Federal Emergency Management Agency, 2000) using a genetic algorithm (GA).

2. LIMITATIONS OF FORCE BASED DESIGN

Current design office methodologies for seismic design of concrete moment frames include forced based methods for calculating equivalent lateral forces and a static elastic analysis. Research has revealed erroneous assumptions in forced based methods and proposes that displacement based methods, due to modelling inelastic systems; result in more reasonable lateral force distributions. In some respects the current FBD philosophy incorporates certain aspects of PBSE. That is current seismic codes require the design engineer to check service level and ultimate displacement demands against code requirements. Similarly, elastic force demands are checked against ultimate plastic capacities. However, these are grossly approximated since an elastic analysis cannot account for inelastic redistribution and higher mode effects are incorporated only by a top level concentrated force with no evaluation of the stiffness and strength of individual floors. Additionally, the final ductility demand is determined by the amplification of service level displacements; albeit with no requirements to insure that the final ductility capacity does in fact match the initially assumed force reduction factor. This implies that a ductility capacity can be assigned to a structural system regardless of its geometry and member strengths (Priestley and Kowalsky, 2000). Additionally, the stiffness of a structure solely determines its displacement response (Priestley and Kowalsky, 2000).

Furthermore, current seismic codes limit inelastic story drifts to 0.02 or 0.025 depending on the initially "assumed" 1st mode period. As a consequence, code drift limits tend to reduce design ductility levels to values significantly less than what can actually be accommodated (Priestley and Kowalsky, 2000). Ultimately, the system will not be able to accommodate the full ductility demand (assumed equal to the force reduction factor by current seismic codes). Thus producing higher than expected seismic forces and possibly leading to either unexpected damage or damage levels in excess of desired. Furthermore, FBD procedures were developed based on the results from early scale model research and current research is noting that those results are not appropriate for determining or predicting the behavior of larger complex systems.

It is the expectation that the proposed DBD procedures will eliminate the fundamental problems inherent in FBD methods by means of reverse engineering. That is the final ductility demand, or damage level, is the starting point of the design process, not the approximate final result. Therefore, the initial system stiffness and respective member strengths are the final design results. It is out of the scope of this document to discuss current FBD procedures.

3. FUNDEMENTALS OF DIRECT DISPLACEMENT BASED DESIGN

In DDBD the multi degree of freedom system is represented by an single degree of freedom system as shown in Fig. 3.1a. Force based seismic design characterizes a structure in terms of elastic, pre-yield properties (initial stiffness, elastic damping), DDBD characterizes a structure by secant stiffness K_{ef} at maximum displacement as shown in Fig. 3.1b. and a level of viscous damping and hysteretic energy absorbed during elastic response (Priestley, 2003). This is based on Substitute structure approach (Shibata and Sozen, 1976).

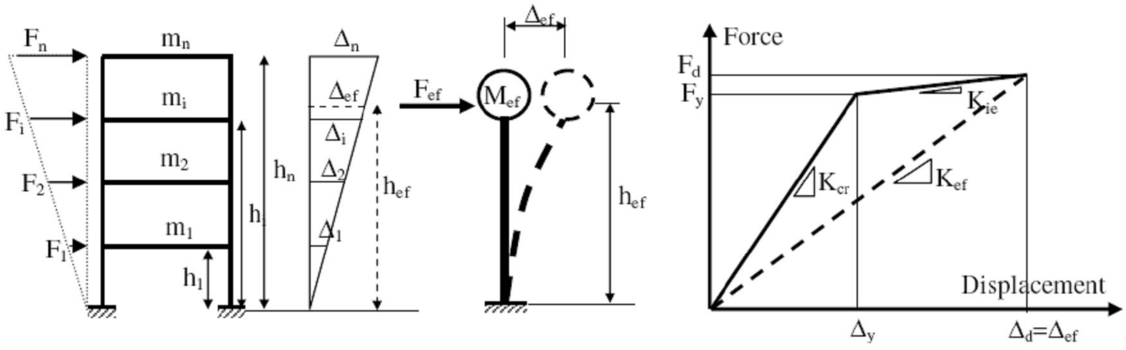

Figure 3.1. (a) Equivalent single degree of freedom system (b) Effective Stiffness

When the displacement profile as shown in Fig. 3.1a. is known then the design displacement Δ_d, can be obtained by using Eqn. 3.1.

$$\Delta_d = \frac{\sum_{i=1}^{n} m_i \Delta_i^2}{\sum_{i=1}^{n} m_i \Delta_i} \tag{3.1}$$

where n is the number of story, m_i is the story mass and Δ_i is the story displacement obtained by the displacement profile.

In DDBD, the nonlinear behavior is represented by an equivalent viscous damping (ξ_{eq}). Using equivalent value of viscous damping representing both elastic and hysteretic energy dissipation, it is possible to solve a simple nonlinear system instead of a nonlinear system. The effective damping depends on the structural system and displacement factor. There are several equivalent approaches. For frame structures Eqn. 3.2. is selected (Priestley, 2003)

$$\xi_{eq} = 5 + \frac{125\left(1-\mu^{-0.5}\right)}{\pi} \tag{3.2}$$

Where μ is the ductility factor.

The effective period (T_{ef}) of the substitute structure can be obtained from displacement a spectrum which is reduced for the determined damping ratio as shown in Fig. 3.2.

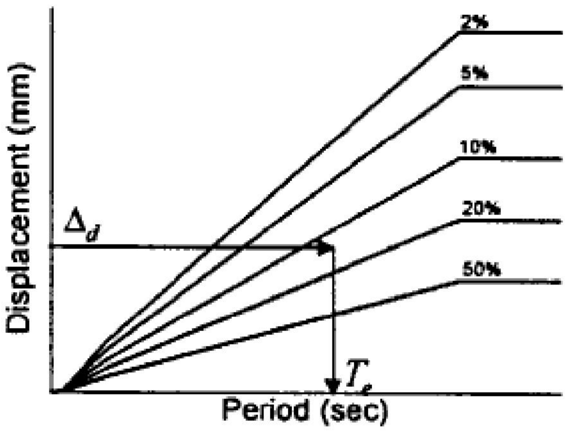

Figure 3.2. Obtaining effective period.

Effective stiffness (K_{ef}) of the equivalent single degree of freedom system at maximum displacement can be found using the equivalent period with Eqn. 3.3.

$$K_{ef} = \frac{4\pi^2 M_{ef}}{T_{ef}^{\ 2}} \tag{3.3}$$

where M_{ef} is the effective mass of the SDOF system and can be obtained using the displacement profile as in Eqn. 3.4.

$$M_{ef} = \frac{\sum_{i=1}^{n} m_i \Delta_i}{\Delta_d} \tag{3.4}$$

As shown in Fig. 3.1b. the base shear of the equivalent SDOF system is given by Eqn. 3.5.

$$V_b = K_{ef}\Delta_b \tag{3.5}$$

With the base shear and the story displacements calculated the story forces can be found by Eqn. 3.6.

$$F_i = \frac{V_b m_i \Delta_i}{\sum_{i=1}^{n} m_i \Delta_i} \tag{3.6}$$

The story forces are proportional to story displacements. The force profile for the system has the same shape as the maximum displacement profile.

In the study the structure will be analyzed under the force vector obtained. In order to be compatible with the substitute structure concept, member stiffness should be representative of secant stiffness at design displacement response for frame system building, beam members will be subjected to inelastic actions and the appropriate stiffness can be given by Eq. 3.7.

$$I_b = \frac{I_{cr}}{\mu_b} \tag{3.7}$$

where I_{cr} is the cracked section stiffness; μ_b is the ductility factor of beam. The columns and the wall will be protected against inelastic actions so that their stiffness can be taken without reduction of ductility. However the wall stiffness will need to be reduced over the lower levels in proportion to expected ductility demand. Plastic hinge will be expected at the base level of the ground floor columns and wall. A modification to the column stiffness must be made for columns ground floor. The most appropriate way to model this in an elastic analysis is to place a hinge at the base level and apply a base resisting moment as shown in Fig. 3.3. These moments are called prefixed moments (Priestley and Kowalsky, 2000).

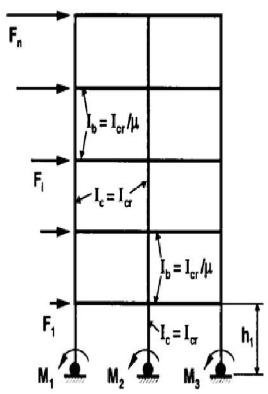

Figure 3.3. Member stiffness for structural analysis

Before a complete structural analysis is done, the base moments must be determined. Column moment (M_C) is a designer choice. But can be calculated using the base shear of the column and the height of the point of contraflexure which is between %55 and %65 of the column height. With a point of contra flexure chosen %60 of the column height, column moment (M_C) can be given in Eqn. 3.8.

$$M_C = V_C(0.6h_l) \tag{3.8}$$

Where V_C is the column base shear force, h_l is the height of the first floor(Priestley, 2000).

Displacement profile is an important step of the DDBD because most of the properties of the equivalent linear system is obtained using displacement profile.

For this research, the estimated inelastic displacement profiles are shown in Table 3.1. (Priestley and Kowalsky, 2000)

Table 3.1. Inelastic displacement profiles

n	Δ_i
$n \le 4$	$\theta_T h_i$
$4 < n < 20$	$\theta_T h_i \left(1 - \frac{(n-4)h_i}{32h_n}\right)$
$n \ge 20$	$\theta_T h_i \left(1 - \frac{h_i}{2h_n}\right)$

where

n = number of stories

h_i = respective story height (in, mm)

h_n = Total frame height (in, mm)

Target interstory drifts, θ_T, are selected to define the desired performance level, or limit state, Corresponding to the respective damage level or desired ductility demand. Based on the selected interstory drift limits and considering the inelastic system displacement shape function at the respective performance levels, the target, or design, displacement profiles can be determined.

4. DESIGN OF REINFORCED CONCRETE FRAMES USING A GENETIC ALGORITHM

4.1. Introduction

Reinforced concrete structures have considerable compressive strength compared to most other materials. In addition to the high compressive strength, reinforced concrete structures are durable, versatile, and have relatively low maintenance cost when compared to steel structures. They also provide good resistance against fire and water damage, and have an excellent potential for a long service life.

Material cost is an important issue in designing and constructing reinforced concrete structures. The main factors affecting cost are the amount of concrete and steel reinforcement required. It is, therefore, desirable to make reinforced concrete structures lighter, while still fulfilling serviceability and strength requirements. In addition to material costs, labor and formwork costs are significant. Beam elements evaluate based on their flexural response considering moment magnification factors due to frame stability. A rectilinear column strength interaction diagram is used to evaluate the feasibility of columns with moment magnification due to slenderness effects. The limitations and specifications of the ACI Code and FEMA-356 formulate as a series of constraints to the discrete cost optimization problem and apply as penalties on the fitness function of the genetic algorithm.

The objective of this research is to design low-cost reinforced concrete frames that satisfy the limitations and specifications of the American Concrete Institute (ACI) Building Code and FEMA-356 using a genetic algorithm (GA). To receive aim an optimal computer program is written in MATLAB software that named DBD-GA. This program recalls OpenSees software. OpenSees was developed at Pacific Earthquake Research Center and is an object-oriented framework for finite element analysis. Structural members have been modeled by using a beam column element based on distributed plasticity fiber element approach.

4.2. Objective Function

The objective of this study is to design reinforced concrete frames that minimize the structural cost. Author defines the basic geometry of both a reinforced concrete beam and a reinforced concrete column where b is the width of the beam or column section, h is the thickness of the beam or column section, and As is the area of the steel reinforcing. In this study, the design variables are the width of the section, b, the thickness of the section, h, the reinforcing steel bar number, and the number of bars or topology of the reinforcement. An advantage of using the rebar number as a design variable is that both the cross-sectional area and the diameter are intrinsic properties. In this case, values associated with a rebar number variable can be used to compute the total cross-sectional area of the steel can be used to compute the total cross-sectional area of the steel reinforcement, As , the flexural capacity of a section, and to determine if a reinforcement pattern is consistent with design geometry. The reinforcement topology variable can define both the number and pattern of reinforcement bars within a section. The mathematical form of the objective function for the design of reinforced concrete frames is

$$\begin{aligned} &Minimize \quad F = \sum_{i=1}^{n_b+n_c} \{C_c b_i h_i + C_s A_{st} + 2C_f (b_i + h_i)\} li \\ &Subject\ to: \ C_1 < 0, C_2 < 0, ..., C_n < 0 \end{aligned} \tag{4.1}$$

where n_b=number of beams in the frame; n_c=number of columns in the frame C_c =cost of the concrete per cubic meter; C_s = cost of steel per cubic meter; C_f = cost of the formwork per square meter ; l=length of the beam or column; and $C_1, C_2, ..., C_n$, are constraint functions based on the specifications and limitations of the ACI Code and FEMA-356. A simple evaluation of the objective function defined in Eqn. 4.1. reveals that for most cases, the costs of the reinforcing steel and the formwork contribute more to the structural cost estimate than the cost of the concrete. However, the combined costs associated with the geometry of the cross section are typically more significant than the cost of the steel reinforcing (Camp, Pezeshk and Hansson, 2003).

4.3. Penalized Objective Function

In engineering optimization problems, it is vital to satisfy performance constraints. in this study, an implementation of linear and quadratic penalty functions is used to account for constraint violations (Goldberg, 1989). The general form of the penalty function is

$$\Phi = F + R_P[\sum_{i=1}^{nel} (\max(\frac{a_i}{a_{all}} - 1, 0)^2)] \tag{4.2}$$

where R_P penalty factor; nel total number of constraints; a_i value of constraint a_{all} allowable value of constraint.

5. DESIGN EXAMPLE (TWO-BAY SIX-STORY FRAME)

A two-bay six-story reinforced concrete frame is selected. The dimensions of the frame are: H=3 m, L_1= L_2=5m where H and L are height of stories and length of spans respectively. Based on FEMA-356 life safety performance level and design drift =0.02 are supposed. Other parameters are given in Table5.1, Table5.2, Table5.3 and Table5.4. also the DBD-GA design results for the beam and column elements are listed in Table5.5, Table5.6 and Table5.7.

Table 5.1 Design Properties for bar and concrete

Fc(Mpa)	Ec(Mpa)	(t/m3)γ	fy(Mpa)	Es(Mpa)
30	25700	2.4	450	200000

Table 5.2 Required parameters for this example

Population size	crossover technique	probability of crossover	probability of mutation	cost of concrete ($\$/m^3$)	cost of steel ($/kg)	cost of formwork ($\$/m^2$)
50	Double point	0.8	0.001	500	8	50

Table 5.3 Beam and Column group numbers

Beam Group Number		Column Group Number		
1 (Stories1-2-3)	***2*** (Stories 4-5-6)	***3*** (Stories1-2)	***4*** (Stories 3-4)	***5*** (Storie5-6)
1	7	13	19	25
2	8	14	20	26
3	9	15	21	27
4	10	16	22	28
5	11	17	23	29
6	12	18	24	30

Table 5.4 Search Space Parameters for Two-Bay Six-Story Frame

		b	h	Number of bars	bar size
column	Min	20	20	4	18
	Max	70	70	8	24
	Increment	5	5	1	2
beam	Min	20	20	4	20
	Max	70	70	12	24
	Increment	5	5	1	2

Table 5.5 Design Results for stories

Floor	Δ_y	μ	ξ	H_{beam}	$\Delta_{dis.profile}$	Force(KN)
Roof	.18	1.35	10.5	.40	.34	210
5	.18	1.35	10.5	.40	.28	176
4	.18	1.35	10.5	.40	.23	142
3	.16	1.52	12.5	.45	. 18	109
2	.16	1.52	12.5	.45	. 12	73
Ground	.16	1.52	12.5	.45	.06	36

Table 5.6 Design Results for substituted structure

$\Delta_{d\ sys}$	$m_{ef\ sys}$	$\xi_{ef\ sys}$	$T_{ef\ sys}$	$K_{ef\ sys}$	$V_{b\ sys}$	k	$M_{b\ sys}$	M_c
.245	161.83	11.53	1.45	3044	746	.65	1454	485

Table 5.7 Optimal Design Results

variables	Beam Group Number		column Group Number		
	1	2	3	4	5
b	35	40	45	45	35
h	40	45	55	50	45
$A_{s\ bottom}$	8 Φ22	7Φ22	5Φ24	5Φ24	4Φ22
$A_{s\ top}$	5Φ22	5Φ22			
Cost	$ 44,291				

6. CONCLUSIONS

This research has proposed an alternate viable seismic design philosophy that inherently eliminates the current restrictions and erroneous assumptions observed in current FBD. Although a direct comparison between the two philosophies was not included, a concrete frame designed in accordance with DBD is more efficient by means of ductility capacity and behavior. Additionally, the design engineer has a better sense of the various degrees of damage. A computer program for designing low-cost reinforced concrete frames using genetic algorithm and direct displacement based design is presented. The DBD- GA design procedure minimizes the material and construction cost of reinforced concrete while satisfying the limitations and specifications of the ACI Code and FEMA-356. A design example is presented to demonstrate the effectiveness and efficiency of the DBD-GA procedure.

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